DOOSAN Document Transmittal

Upper Trishuli-1 HEP (216MW)

Date	DT No.:	SHEET
2022-06-07	UT1-HEP-DHI-PC-D-0034	1 OF 1

DOOSAN NEPAL BRANCH OFFICE	Address & Contact No. : Four Square Building, Naxal, Kathmandu, Nepal (P.O Box : 1387) + 977 1 692 3014	Project No. H20002	DOCUMENT DISPATCHED BY Courier Service ☐ Messenger ☐ Post-Air Freight ☐ Document System or E-mail ■ By hand ■
DOCUMENT DISTRIBUTION	■ POWER CHINA (Subcontractor) ☐ Doosan's Site Office		
SUBJECT	Review Note RN-0135_Stability Calculations for the Surrounding Rock of Headrace Tunnel		

S. N.	REVIEW NOTE NO.	REVIEW STATUS	DWG. / DOC. NO.	REV.	DWG. TITLE / DOC. DESCRIPTION	REMARK
1	RN-0135	AN	UT1-C-150-CVL-DC-54001	0A	Stability Calculations for the Surrounding Rock of Headrace Tunnel	
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						

Received by :

Date :

Legend of Review Status
A : APPROVED
AN : APPROVED AS NOTED
RR : RETURN FOR RESUBMISSION
FI : FOR INFORMATION

DHI STAMP

Doosan Heavy Industries & Construction Co. Ltd. DOOSAN Branch Office Nepal

Issued by DHI

Youngsoo Son

Project Manager

Email: youngsoo.son@doosan.com
Doosan Heavy Industries & Construction Co., Ltd.

Discipline : Civil

UPPER TRISHULI-1 HEP (216MW)			
Client	**Nepal Water and Energy Development Company Private Limited**		
Owner's Engineer	**Tractebel Engineering GmbH / Jade Consult**		
REVIEW NOTE			
Contractor	**Doosan Enerbility**		
Title of the Document	Stability Calculations for the Surrounding Rock of Headrace Tunnel		
Document No.	UT1-C-150-CVL-DC-54001	Revision	0A
Date of Documents	31.05.2022	Received Date	31.05.2022
Transmittal Form No.	UT1-HEP-DHI-D-0249		
Previous Review Date/Status	N/A	Prev. Review Note No.	N/A
Review Note No.	**RN-0135**	**Present Review Date**	**06.06.2022**
		Present Review Status	**AN**
A: Approved AN: Approved as Noted RR: Returned for Resubmission FI: For Information, only			

General Comments

The calculation report in hand addresses the constructability of the Headrace Tunnel with the shown geometry and support, only. It is not a calculation report that can serve as a design document for a low-pressure water tunnel. There are no considerations done regarding the design of a waterway that is from time to time dewatered.

Therefore, any approval given refers to the constructability and stability of the underground structure as such, but it does not refer to the design of this tunnel serving as waterway in the UT-1 Project. It is the solely responsibility of the Contractor to proof, with an appropriate design, that this underground structure with the suggested geometry and support can serve as waterway in the project.

The current concept shows a fully lined tunnel for all rock classes (shotcrete and shotcrete / reinforced concrete liner, unperforated). During operation several scenarios must be considered in the up-coming design of this tunnel as a low-pressure water tunnel:

1. *Groundwater table below the tunnel*
   In this case for all shotcrete lined (=unperforated) sections of the tunnel, which are in engineering terms permeable with time during operation through many factors, a groundwater table will build up around the tunnel that is equivalent to the permanent water pressure in the tunnel. During dewatering (= rapid as compared to operating time) a water pressure that is equivalent to the tunnel internal operating pressure will act on the liner. The shotcrete liner must be designed accordingly. This case is, however, generally avoided by perforating the tunnel liner (= highly permeable liner). The loss of water from the tunnel, acceptable or not, depends of course on the permeability of rock mass.
2. *Groundwater table at or above the tunnel, groundwater pressure **less** than tunnel internal pressure.*
   The same applies as for item 1.
3. *Groundwater table above the tunnel, groundwater pressure **higher** than tunnel internal pressure.*
   In this case, during operation, groundwater will percolate into the fully shotcrete lined tunnel, which is acceptable. Full groundwater pressure will act onto the liner when dewatering.

For reinforced concrete lined tunnels, the situation is different. Due to the low internal pressure during operation, the tunnel liner will not crack; there is different internal and external pressure, depending on the location of the groundwater table. During dewatering the external pressure will act on the concrete liner, fully.

This makes it clear that the knowledge of the external water pressure (= groundwater table) is essential for the design of a low-pressure tunnel. However, prior to construction, any estimate regarding groundwater level with respect to the tunnel elevation is purely speculative for the 10 km long Headrace Tunnel at UT-1 Project. Consequently, it is necessary to determine the expected permanent groundwater table when opening the underground excavation and provide several alternatives for the support of the Headrace Tunnel.

In the following few comments are given on details of the calculation report. They are regarding the constructability and stability of the Headrace Tunnel as a tunnel, but this does not refer to the usability/stability as a waterway. The designer shall be aware that there are many reported cases in hydropower engineering, where low-pressure tunnels have collapsed during dewatering, often over hundreds of meters or even kilometres due to inappropriate design as a waterway. This shall be avoided in the UT-1 Project.

Layout of headrace tunnel refers to drawing No. "UT1-C-150-CVL-DG-54001". The excavation sections and corresponding final proposed supports of each rock class are shown in Figure 1.1~1.6.

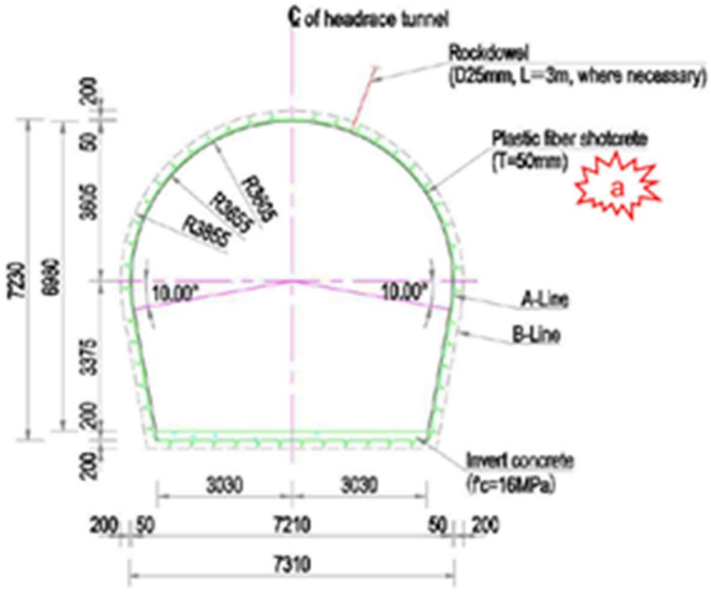

Figure 1-1 Typical section of headrace tunnel (rock class I)

(a) There is no reason to apply 50 mm shotcrete in Class I rock. Why to replace solid rock of high strength with comparatively low strength shotcrete? This is independent whether the liner will be perforated, or not. Omit.

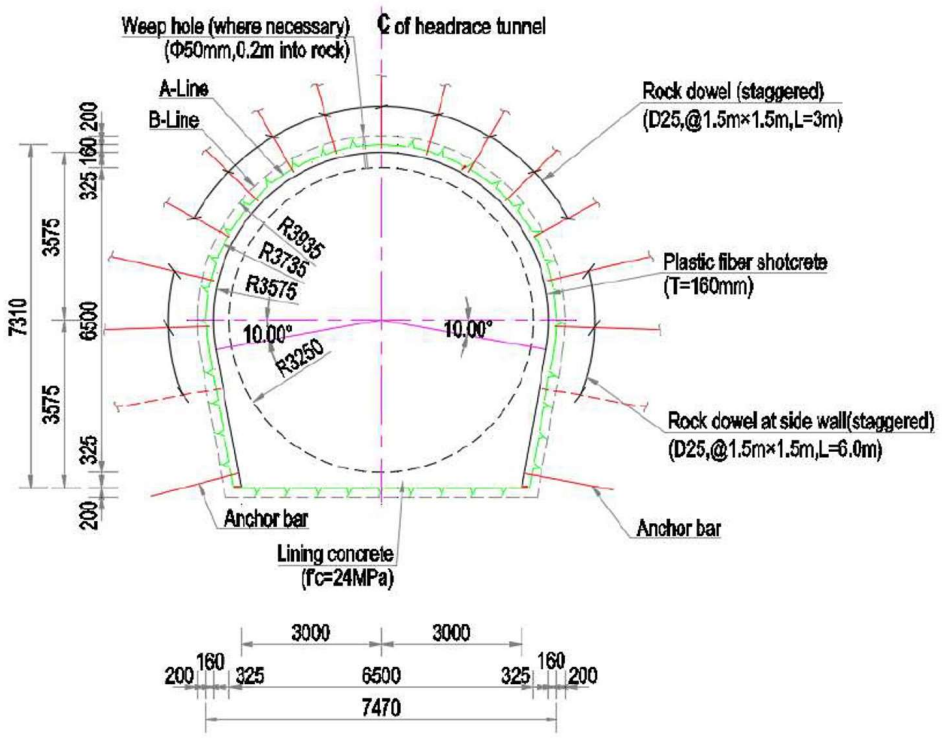

Figure 1-4 Typical section of headrace tunnel (rock class IV, buried depth below 240m)

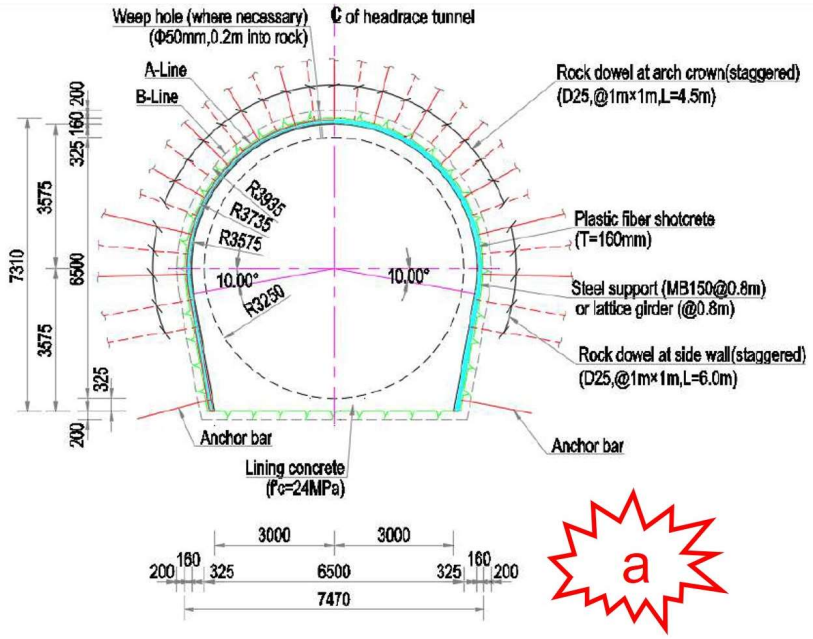

Figure 1-5 Typical section of headrace tunnel (rock class IV, buried depth between 240m and 458m)

(a) The reviewer questions the geometry given in the drawings. The same excavation size is given for the tunnel without and with steel rib support. The steel rib shall not be embedded in the reinforced concrete liner. Revise.

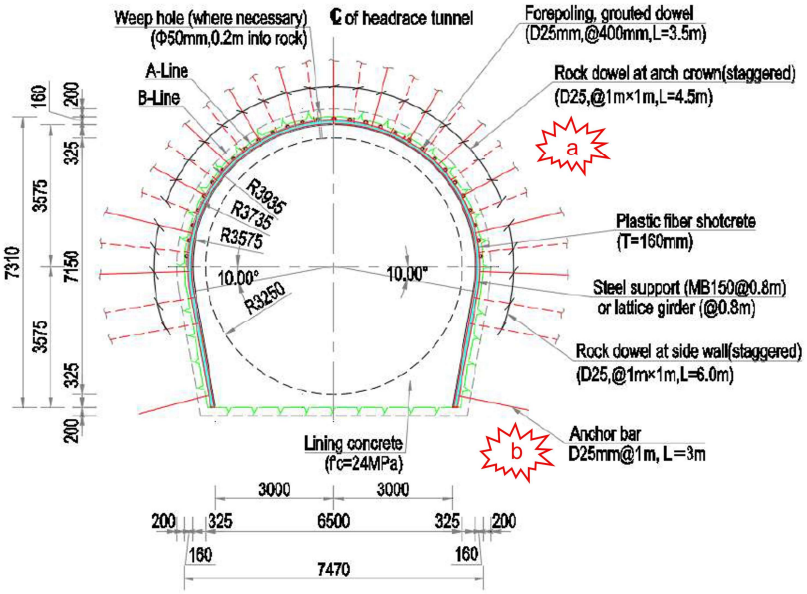

Figure 1-6 Typical section of headrace tunnel (rock class V)

(a) Indicate ground improvement measurements as indicated in the text and specified further below. Provide several improvement suggestions (grouting, canopy tubes etc.) and specify in a table.
(b) Provide provision for steel invert strut.

1.2 Design Objective

a

This calculation is aim to verify the proposed supports include rock dowels, shotcrete and steel support be effective and feasible; ensure stability of tunnel and safety of construction activities.

1.3 References

(1) Contract documents, Employer's Requirement
(2) Tunnels and Shafts in Rock (EM 1110-2-2901)
(3) Layout drawing of headrace tunnel: UT1-C-150-CVL-DG-54001
(4) Excavation and support drawing of headrace tunnel: UT1-C-150-CVL-DG-54002
(5) High Strength Deformed Steel Bars and Wires for Concrete Reinforcement Specification (Fourth Revision) (IS 1786-2008)
(6) Building Code Requirements for Structural Concrete and Commentary (ACI 318)
(7) Report on Hydraulic Fracturing Tests for Rasuwagadhi Hydroelectric Project

b

c

(a) Clearly specify the design purpose. Which requirements have to be fulfilled for a water tunnel? This is the essential chapter in this report.
(b) This document shows copies of the confirmed tunnel profiles and a longitudinal section of the tunnel. Specify which document refers to each other. It should be clear, which document is the initial document, and which is approved with the calculation report in hand.
(c) See comment below.

Rasuwagadhi Hydroelectric Project is 25 kilometers away from Upper Trishuli-1 HEP. The in-situ field stress test results are of reference significance for that of Upper Trishuli-1 HEP. According to Table 5-4 a Report on Hydraulic Fracturing Tests for Rasuwagadhi Hydroelectric Project". Azimuth angle of First and third principle field stress is 18° and 285°. Second principle field stress is in gravity direction.

(a) It is understood that the designer wants to support the assumptions on in-situ stresses, which is appreciated. However, as the alignment of the Headrace Tunnel is quasi parallel to the maximum principal stress σ_1 in the region – or at an acute angle, only – it is fully justified to assume k = 1.0. There is no further justification or derivation required. In the plane-strain calculations, the maximum principal stress σ_1 is the "out-of-plane stress" and is hence irrelevant.

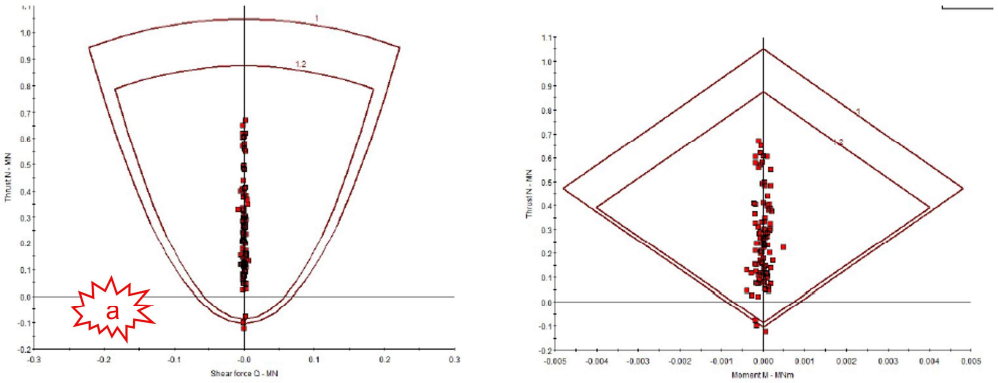

Figure 6.2-4 Bearing capacity of shotcrete (Rock Class II, 608m)

(a) The results show that the shotcrete liner does not satisfy the design requirements. This should be stated.

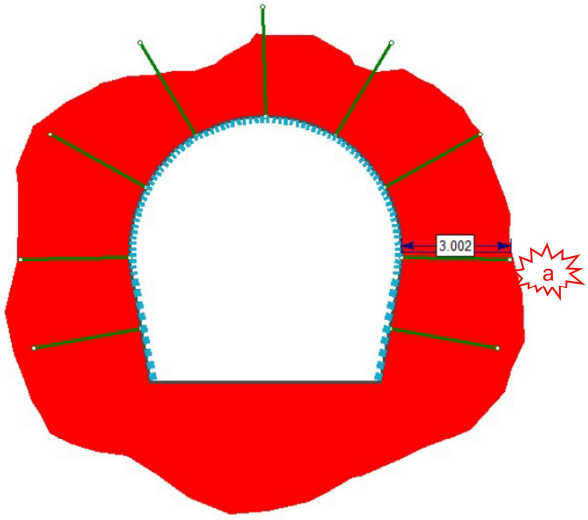

Figure 6.3.2-1 Surrounding rock plastic zone (Rock Class III) (buried depth 608m)

(a) The length of dowels needs evidently be adjusted. Revise respective drawing.

6.4.4 Rock class IV (Buried depth 458m, Prop supports)

It is planned and proposed by conducting pre-consolidation grouting to improve the rock mass properties. After consolidation grouting, surrounding rock properties of 1-5m depth is radius range are as follow: a

Table 6-1 Mechanical parameters of rock class IV after consolidation grouting

Class of surrounding rock	Elastic modulus of rock mass	Rock mass/rock mass shear (shearing) strength				Uniaxial compressive strength of rock mass	Uniaxial compressive strength of intact rock	Tensile strength of rock mass
		Peak		Residual				
	Em (GPa)	*C* *(MPa)*	*Φ* *(°)*	*Φr* *(°)*	*Cr* *(MPa)*	UCS rm (MPa)	INTACT UCS r (MPa)	σtm (MPa)
IV (Before consolidation grouting)	2.3	0.5	35	27.5	0.15	1.9	60	0.19
IV (After consolidation grouting)	5	0.8	40	30	0.25	1.9	60	0.19

b

(a) Specify how grouting shall be done to achieve the respective improvement in rock mechanics parameters.

(b) Where do these "improved" parameters come from? Specify. If they are pure estimates, then state accordingly. Better make reference to an actual case in literature.

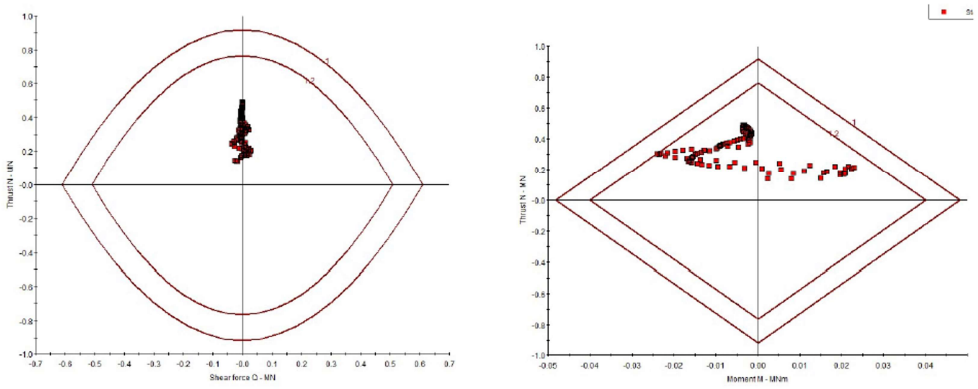

Figure 6.4.6-5 Bearing capacity of steel support (Rock Class IV, 608m)

For rock class IV with buried depth of 608m, by increasing rock dowel length, strength and thickness of shotcrete, and steel rib MB 150@0.8m, the side wall rock dowel length of 6m is less than plastic zone depth of 6.6m. Crown displacement is 150mm, which has exceeded 120mm. The maximum rock dowel stress is 331.3MPa, which is less than the allowable value of 333MPa. The Bearing capacity of shotcrete cannot meet the requirement of safety factor of 1.2. a

(a) What is the conclusion from this statement? Ground improvement, heavier support? Specify.

7 Wedge Analysis

a

7.1 Discontinuity Input Data

According to the site investigation, there are three group joints are distributed along the headrace tunnel alignment, as shown in Table 7-1. The Mechanical parameter of joints is shown in Table 7-2. The stereographic projection of these joints in rock mass is shown in Figure 7-1~7-3.

(a) The tunnel is longer than 10 km. Discontinuity analyses should be based on reliable data for each section of the tunnel, but cannot currently be done appropriately with the available data. Therefore, this chapter is weak with respect to the design of the tunnel. It should be stated how this work is updated in practise during tunnel construction to satisfy appropriately the design requirements.

6) With buried depth of 608m of proposed supports (arch crown and side wall rock dowel length increased to 4.5m and 6m respectively, shotcrete strength is increased to 25MPa and thickness is increased to 160mm, steel rib MB150@0.8m, and it is planned and proposed by conducting pre-consolidation grouting to improve the rock mass properties), the calculation results cannot fully meet safety requirements. The side wall rock dowel length of 6m is less than plastic zone depth of 6.6m; the maximum crown displacement is 150mm, which exceeds 120mm; the maximum rock dowel stress is 331.3MPa, which is less than the allowable value of 333MPa; The Bearing capacity of shotcrete cannot meet the requirement of safety factor of 1.2. The proposed supports are relatively strong from view point of construct ability but shotcrete bearing capacity still cannot meet safety requirement.

a

(5) This calculation mainly covers rock class I~IV, to rock class V, it is planned and proposed by conducting pre-consolidation grouting to improve the rock mass properties, additional support measures such as forepoling, canopy tube may be considered.

b

Both calculations, for Class IV (608 m), (a) and Class V (b) have shown that the tunnel cannot built with the suggested support. This is understood, and the experienced tunnel designer/engineer knows that tunnels can be built in such bad conditions. A detailed solution/conclusion must be provided and specified how the tunnel can be built under such conditions.

DISCLAIMER

Any approval will not release the Contractor from any of his obligations under the Contract.

Dr. U. Glawe